VOLUME 79 SEPARATE No. 244

PROCEEDINGS

AMERICAN SOCIETY OF CIVIL ENGINEERS

AUGUST, 1953

STRUCTURAL OBSERVATIONS OF THE KERN COUNTY EARTHQUAKE

by Henry J. Degenkolb

STRUCTURAL DIVISION

(Discussion open until December 1, 1953)

Headquarters of the Society
33 W. 39th St.
New York 18, N. Y.

PRICE $0.50 PER COPY

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This paper was published at 1745 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-ninth Street, New York 18, N. Y.

STRUCTURAL OBSERVATIONS OF THE KERN COUNTY EARTHQUAKE

Henry J. Degenkolb, Member, A. S. C. E.*

On July 21, 1952, an earthquake of considerable magnitude shook a major portion of Kern County. This earthquake was caused by a sudden displacement along the White Wolf Fault extending from Wheeler Ridge, past Arvin, and between Caliente and Tehachapi as indicated on the map shown in Figure 1.

According to the Gutenberg Richter Scale, this earthquake was of magnitude 7-1/2, releasing $10^{17.7}$ foot pounds of energy. This may be compared to the Long Beach Earthquake of 1933 which had a magnitude of 6-1/4, releasing $10^{15.4}$ foot pounds of energy and the San Francisco Earthquake of 1906 with a magnitude of 8-1/4, releasing 10^{19} foot pounds of energy. Another commonly used measure of earthquake intensities, the Modified Mercali Scale, is based on average observed damage and human reactions. On this scale, where the San Francisco Earthquake was of magnitude 10, and the Long Beach Earthquake was of magnitude 9, the July 21 shock is rated at VIII at Tehachapi, Bakersfield and Arvin with a maximum of XI near Caliente.

Notwithstanding the above measures of magnitude, it is the general concensus of opinion of the engineers familiar with the results of both earthquakes, that the effect on buildings in Tehachapi was only about 75% as great as that of the Long Beach Earthquake. Some engineers feel that Arvin was shaken as hard or harder than Tehachapi, but the available evidence is contradictory and it is hard to draw any definite conclusions.

These inconsistencies emphasize the fact that at the present time there is no adequate engineering scale of intensities that measures the amount of destructive "force" that causes damage to structures. While the Richter Scale measures the total energy release, it takes no account of the area over which this energy is diffused. A large energy release, scattered over a large area, may have less damaging "force" at its maximum intensity than a relatively small energy release, if the smaller release is concentrated over a smaller area. On the other hand, the Modified Mercali Scale is deficient from this point of view in that it generalizes from the effects on structures. The damage in Tehachapi was greater and much more spectacular than that at Arvin. However, Tehachapi is an older town with many old style masonry buildings particularly vulnerable to earthquake damage, while Arvin is comparatively young with a higher percentage of newer buildings built by men who have had experience in the construction of earthquake resistant structures. In Caliente, with only small frame buildings, comparatively little damage was noted in spite of its high rating. So it is easily seen that a magnitude scale based on observed damage must be indefinite, to say the least.

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*Chief Engineer, John J. Gould, Consulting Engineer, San Francisco, California

It is impossible in a paper of this type to cover exhaustively all of the effects of this earthquake, so I must limit myself only to buildings in the areas of maximum damage and to certain of those buildings which seem to me to illustrate interesting points.

The statement has been made that no new lessons have been learned from this earthquake. If this is meant to apply to the spectacular damage featured in the newspapers, this is undoubtedly true. The older style masonry buildings, built with poor mortar and not tied together, fared badly as they have in all previous major earthquakes. There is little to learn from such failures except to emphasize to the lay public how inadequate and unsafe it is. However, this earthquake has "tested" more buildings constructed under the newer codes than has any previous earthquake, and there is much to learn from the behavior of these buildings.

In general, those buildings that were designed and built with some conscious thought of lateral force resistance—those that met code requirements in this regard—performed very well. The serious damage was practically confined to those buildings, usually older, which had no provisions whatsoever for lateral resistance. It is also interesting to note that of the newer buildings that were not adequately designed for lateral forces, less than average damage usually occurred. It is probable that many construction techniques, learned from constructing earthquake resistant buildings in the general area, were unconsciously applied to buildings not specifically so designed.

Before going to a more detailed examination of some of the damage, it is well to mention one fact that impressed itself upon all observers. The construction of our buildings must be supervised. Many items of damage were caused by inferior materials and workmanship, or by not following the design drawings. Time and again it was found that damage was either caused or aggravated by the omission of anchors, or ties that had been intended by the designer but were omitted in the field. Engineers must realize that a client is constructing a building, not a set of plans. The safety of the public is not protected by a nice set of design calculations if the results are not attained in the field construction.

One approach to studying the performance of structures in an earthquake, is the relationship of their performance as compared to rigidity. Certain flexible structures performed very satisfactorily in this earthquake in spite of the fact that they had little or no bracing. Outstanding among this group of buildings are various timber buildings including warehouses, corrugated iron buildings with either wood or steel framing, and steel structures such as service stations (Figs. 2 and 3). It is interesting to speculate that the man who notified the outside world that Tehachapi had been leveled by the earthquake, may have phoned from an undamaged service station. The Clark Hotel (Fig. 4), an old frame board and batten building, was undamaged; although in the same block the Juanita Hotel (Fig. 5) —an old style masonry building—was badly wrecked killing one of its occupants.

The vast majority of the timber residences were undamaged. Frame structures of all kinds such as churches, schools, stores, etc., had only minor damage or no damage at all, even where there was no apparent lateral force design. The Town & Country Market (Fig. 6) is a wood frame and stucco building that lost its plate glass windows and had minor plaster cracks. Except for this, it can be said that the building was essentially undamaged. The store is 75' x 70' in plan with a 12'-0" ceiling height (Fig. 7). It uses 3 timber trusses with rod bracing at the lower chord and horizontal roof sheathing. When the truss manufacturer designed the rod bracing, he evidently assumed

that there would be resisting elements front and rear. However, there is no front resisting element because there are pipe columns and only simple connections to the beams and so there is considerable torsion on the building. As constructed, it can be figured to resist less than 6#/sq. ft. wind or about 5-1/2%G on dead load only. It is no wonder that the plate glass windows broke. However, in spite of the design deficiencies, the store was open for business shortly after the earthquake.

At the other extreme, we have the heavy, rigid, unreinforced masonry buildings which often suffer severe damage. Houses constructed of adobe with their heavy, weak materials were badly wrecked (Fig. 8). The Cummings Valley School (Fig. 9), built in 1910 of practically unreinforced concrete, was destroyed.

The Tehachapi State Prison for Women, built in 1932 and not designed for lateral forces, consists of a group of two story reinforced concrete buildings with steep pitched tile roofs on wood framing (Fig. 10). An exterior view of these buildings would indicate little damage except for the loss of some chimneys and some cracking, but the interiors show badly cracked tile partitions and a general shambles of the timber roof framing (Figs. 11 and 12). By the standards of present day lateral force design, the timber framing was particularly poor, using inadequate or no ties, and it is not at all surprising to see the amount of damage that occurred. Present codes, however, assign certain shear values for using tile partitions and other similar brittle elements, and on this basis the shear resistance of many partitions in the top floor would approximately meet present Uniform Building Code requirements due to the fact that these partitions were spaced quite closely to provide small rooms (Fig. 13). These partitions, however, were badly cracked (Fig. 14). This observation raises the question as to the adequacy of normal static shear tests, especially of brittle, weak materials to establish safe resistance values for earthquake loads. Present codes show an increasing tendency to permit use of these brittle materials such as plaster, plaster substitutes, hollow tile, weak deck fills, etc., as lateral force resisting elements by assigning allowable stresses for their use. At present, these values are based on static tests, and observations indicate that their performance in actual construction may not meet these expectations.

The performance of these buildings particularly illustrates the necessity of properly tying together various building elements. Where certain key connections are weak, as were the wood to concrete connections in this case, their failure permits a battering action to occur that invariably causes considerable damage. It would have cost very little to make these buildings adequately earthquake resistant by a more judicious choice of construction materials, supervision of field construction, and most important of all, by an adequate engineering design that would result in the building acting as a unit. Without these three necessities, I doubt that high earthquake coefficients as required by present codes, would have appreciably improved the performance of these buildings.

The Arvin High School (Fig. 15) is an excellent example of a building that is quite rigid, met code requirements for California Schools and, in general performed excellently. This large school, located comparatively close to the fault line, was evidently shaken quite severely. One shear wall suffered some damage by cracking (Fig. 16). This wall was constructed of grouted brick masonry, and according to code standards, was not highly stressed. However, field examination and later core samples indicated that the workmanship on this wall was poor, resulting in voids in the grout and inadequate bond to the reinforcing steel. The aftershocks of the earthquake caused this crack to

open somewhat and the loss of rigidity then permitted interior partition damage in this portion of the building.

The only other large school structure of recent construction, located in the area of greatest damage was the gymnasium of the Tehachapi Valley Union High School, located in Tehachapi (Fig. 17). This is a concrete block structure with a complete reinforced concrete frame with a wood roof on steel trusses. It was built during the war, designed in conformance to the School Code requirements of California. All concrete block walls were reinforced, and parapets were made of reinforced concrete made integral with the bond beams supporting them. Because of the roof monitor and the skylights reducing the strength of the roof diaphragm, rod bracing was used at the lower chord of the trusses. The rod bracing was specified to be tightened to 1000# tension, but this was evidently not done. In general, the rods were 1 Ø except at the corners where 1-1/4" Ø were used.

The building was undamaged except that the threads were stripped from the 1-1/4" Ø rods. Later examination showed that the 1-1/4" Ø rods had been threaded with 1-1/8" Ø dies where they entered the 1-1/4" Ø turnbuckles, providing a very weak joint, and again indicating the importance of adequate field supervision.

The concrete block warehouse shown in Figure 18 is constructed of unreinforced concrete blocks with 12 x 12 reinforced concrete columns and tie beams and a wood sheathed roof on steel trusses. There evidently was no effective designed bracing, and the end wall gables were thrown out since there was absolutely no connection at the roof (Fig. 19). Beyond this, however, there was no damage. The floor was at loading platform height with the columns tied to it, so evidently the columns cantilevered above the floor and braced the building sufficiently enough for this earthquake without suffering damage. Although we could not call this a flexible building, the flexibility due to the bending of the columns undoubtedly reduced the forces exerted on the building.

The building for the Tehachapi Supply Company (Fig. 20) was designed and built in 1946 during the material shortages. This building is 40' x 100', using concrete block walls with poured concrete columns and bond beams. Except for the first bay, which is timber, the rest is corrugated iron and there is no bracing as we ordinarily understand it (Fig. 21). However, some resistance is developed by the columns cantilevering out of the footings. At the base of the column, the dowels from the footings are 1/2" Ø although there are 3/4" Ø in the column. When the columns acted as cantilevers, they failed at the base (Fig. 22). There was little or no supervision of construction and changes were made from the design drawings. Dowels to parapets and bolts from wood roof to masonry were omitted while framing details were changed. As constructed, the building as a whole would "figure" for about 4%G or a 6# wind. In spite of the poor design and the poor construction due to lack of supervision, this building is usable and was open for business right after the earthquake. When the columns shattered, they permitted the building to act as a flexible structure and so prevented further damage, which could be interpreted as an interesting application of limit design. The cost of repairs for broken front windows, parapets, tops of front columns and column bases was not great to bring it back to its original condition, but it would have been expensive to bring it up to any code requirements.

In the down town area we have an old brick building with a second floor auditorium that completely collapsed (Fig. 23). The auditorium roof is supported by the piano and seats.

Across the street is the recently built Bank of Tehachapi (Fig. 24). This

building is 56' x 73' with a long low appendage at the rear. It has 7" reinforced concrete walls with steel bowstring trusses providing a 14'-0" ceiling (Fig. 25). There was a wood roof using straight sheathing with a rod bracing system. At the rear of the bank this long appendage had a diagonal sheathed roof with no joint anchors. Based on the Uniform Building provisions, this portion had a lateral load of 200#/ft. on a 50' span, the diaphragm being 14' deep which gives an end shear of 360#/ft. The front is of wood and stucco with an inverted concrete bent taking the lateral loads. According to Uniform Building Code provisions, the rods are stressed to 37,000# per square inch with rather inefficient details for the rods. There was little damage except to the front and to the parapets which fell off (Fig. 26). Note that while the parapet was reinforced with 1/2" Ø @ 3'-0", there were no dowels for the portion of the parapet that broke off.

The Beekay Theater (Fig. 27) in the center of the damaged area was undamaged except for a small crack in the wall on the far side. This building is 43' x 75' with 8" reinforced concrete walls between 12 x 12 columns and bond beam (Fig. 28). These supported 3 bowstring timber roof trusses, wood purlins and straight wood sheathing. It has 5/8" Ø rod bracing in the plane of the lower chord but this ends at the lobby stud wall. The so called plans of this building were very meager. For such a major building element as this wall, there was absolutely on information on the drawings and the plaster prevented a determination of the construction below the ceiling. Assuming the wall to be an adequate shear resisting element (which I personally doubt), the rod bracing is good for about 11%G on dead load only with a shear in the wall of about 350#/ft. The calculated deflection due to the rod system is about 1", assuming rods tight, and neglecting such items as bolt slip and deformations in the shear walls. The maximum permitted by one proposed formula is .8" and must include these effects.

To me the most interesting building in the area was this Auto Sales Agency in Arvin (Fig. 29). Designed and closely supervised by both an architect and an engineer, it suffered the damage of only two cracked panels of glass and a small crack in the masonry over the side door. The framing (Fig. 30) is very simple, using steel beams and columns, a metal roof deck acting as a diaphragm, and unreinforced concrete block walls as resisting elements. However, all the basic elements of lateral force resistance are present and the structure is well tied together, and the results achieved were outstanding, proving that this type of modern architectural design can be made safe for earthquakes.

From the variations in damage and response in the different buildings illustrated above, we can see one basic reason for the conflicting conclusions of different observers of earthquake damage. Our present building codes now require the same coefficients for all structures regardless of rigidity, the construction materials, or type of framing. Requirements of analysis and design are given in only the most general terms, many of which are subject to different interpretations by different engineers and also, I am afraid, in different localities. Each engineer observer views the earthquake damage in the light of his own experience and his individual interpretations of code requirements, and it is not surprising to find such a wide divergency of expressed views.

In view of this divergency, it is most surprising to find the large areas of agreement among engineers who are familiar with earthquake damage in the final result of structural design. To the average engineer, a building code is becoming almost a handbook of design by giving definite loads, allowable stresses and required details. It gives moment coefficients, flat slab coeffi-

cients, hook requirements, minimum nailing, locations of critical shear and definite formulae for almost everything. This detailed approach is not possible in earthquake design, and from a professional viewpoint would not be desirable if it were. In earthquake design, the basic understanding of tying a building together to accommodate an erratic notion is of far greater importance than detailed code requirements. As a result of this fact, we generally find that in the final structure there is considerable agreement between experienced engineers even when operating under the different coefficients required by various codes and a satisfactory structure is attained. For inexperienced engineers following the code as a handbook, these results often may not be attained. It has often been said, and it is only too true, that a thoroughly unsafe building may need code requirements entirely, while a building that will not "figure" may be quite safe. It should be the endeavor of all engineers to bring our code requirements to a common basis that expresses, as far as possible, the experience derived from actual earthquakes.

From the above observations it might be possible to draw certain conclusions as follows:

1. To construct earthquake resistant structures, the prime requisite is to provide for adequate engineering services both in the design and in the field supervision. Neither portion of the service is adequate alone. Too many engineers feel that if the design is adequate, the field construction will take care of itself. Experience has proven that nothing could be further from the truth.

2. It is of prime importance that the various components of a structure be adequately tied together so that it will act as a unit.

3. A blanket requirement for high earthquake coefficients is not sound. Where only the protection of the public is concerned, and for certain types of structures, especially the inherently flexible types, rather modest coefficients are adequate and are in the public interest due to the construction savings involved.

4. Where expensive finishes or rigid interior partitions must be protected, it may be sound economy to provide a rigid bracing system with consequent higher earthquake coefficients. This requirement must be clearly differentiated, however, from the requirements to protect the public safety.

5. In the final analysis, the producing of safe and adequate earthquake-resistant structures for the public must lie basically with the experience and knowledge of the designing engineer rather than with detailed building code requirements. The engineer who knows their capabilities and limitations, must choose the proper materials for each specific job. The engineer must connect them together adequately to provide a coherent unit with sufficient lateral resistance to suit the requirements at hand and yet maintain strict economy for the client. At present the problem is too complex for the provisions of a "handbook" building code that dares not discriminate between various materials and must accommodate its minimum provisions to materials and framing types used in their least suitable manner. Inevitably, a large portion of the consequent requirements are unjustified and not in the public interest.

In closing, I wish to acknowledge the assistance and cooperation of William Cloud of the U. S. C. & G. S. in furnishing certain data. Much of the data from original design drawings was gathered by Gordon Dean and Tom Wosser from the office of John J. Gould, Consulting Engineer.

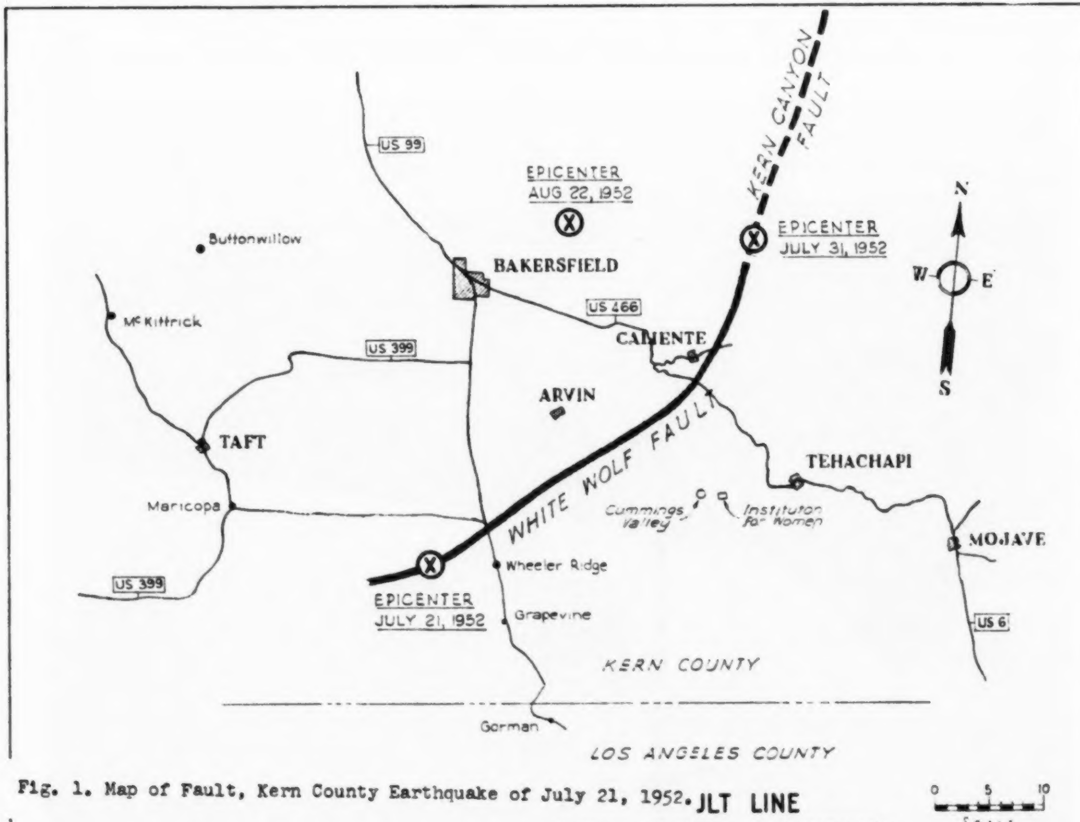

Fig. 1. Map of Fault, Kern County Earthquake of July 21, 1952.

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KERN COUNTY EARTHQUAKE

Fig. 2 Corrugated Iron Warehouse, Tehachapi. Little damage except opening of vertical joints of corrugated iron near base.

Fig. 3 Corrugated Iron Warehouse, Tehachapi. Undamaged, and used as Red Cross Shelter after earthquake. Completely unbraced, light wood framing.

Fig. 4 Clark Hotel, Tehachapi. This board and batten wood building is in very poor condition but was undamaged by earthquake.

Fig. 5 Juanita Hotel, Tehachapi. One guest was killed in this old style masonry 2 story building that was very badly wrecked.

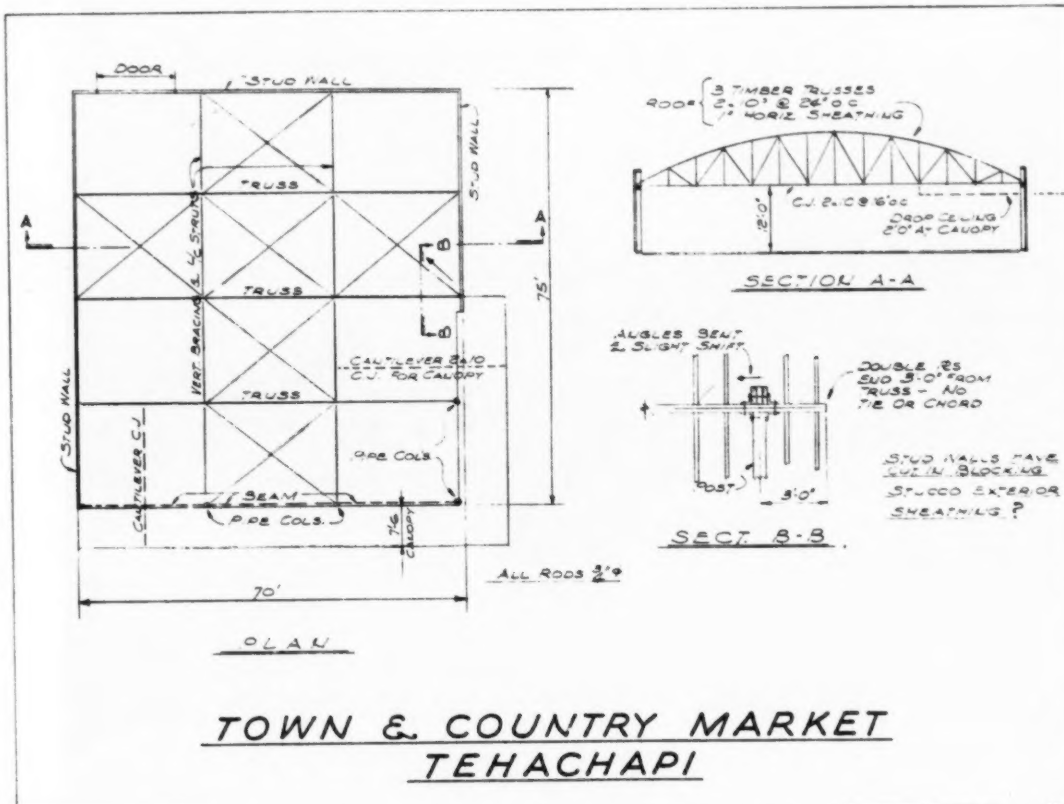

Fig. 6 Framing of Town and Country Market, Tehachapi.

Fig. 7 Town and Country Market. Damage to this frame and stucco store with inadequate bracing was limited to broken windows and cracked plaster. Store was open for business shortly after earthquake.

Fig. 8 Adobe House, Tehachapi. All adobe houses in Tehachapi were badly shattered.

Fig. 9 Cummings Valley School, built in 1910 of practically unreinforced concrete was a total loss.

Fig. 10 Tehachapi State Prison for Women. Exterior view of typical building.

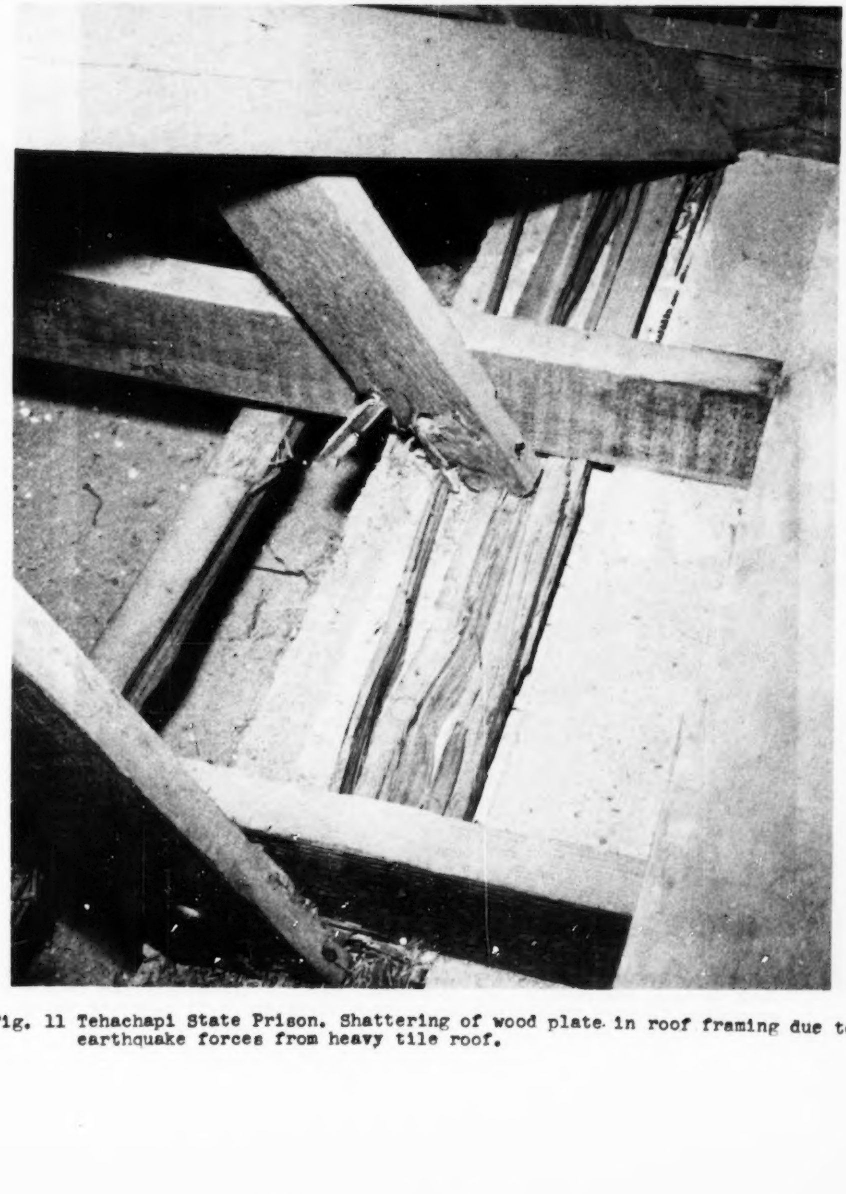

Fig. 11 Tehachapi State Prison. Shattering of wood plate in roof framing due to earthquake forces from heavy tile roof.

Fig. 12 Tehachapi State Prison. Wracking of wood roof framing due to earthquake. Wood members are poorly tied together. Only 2 - 1/2"Ø bars can be seen extending through forty foot long crack in exterior 8" concrete wall.

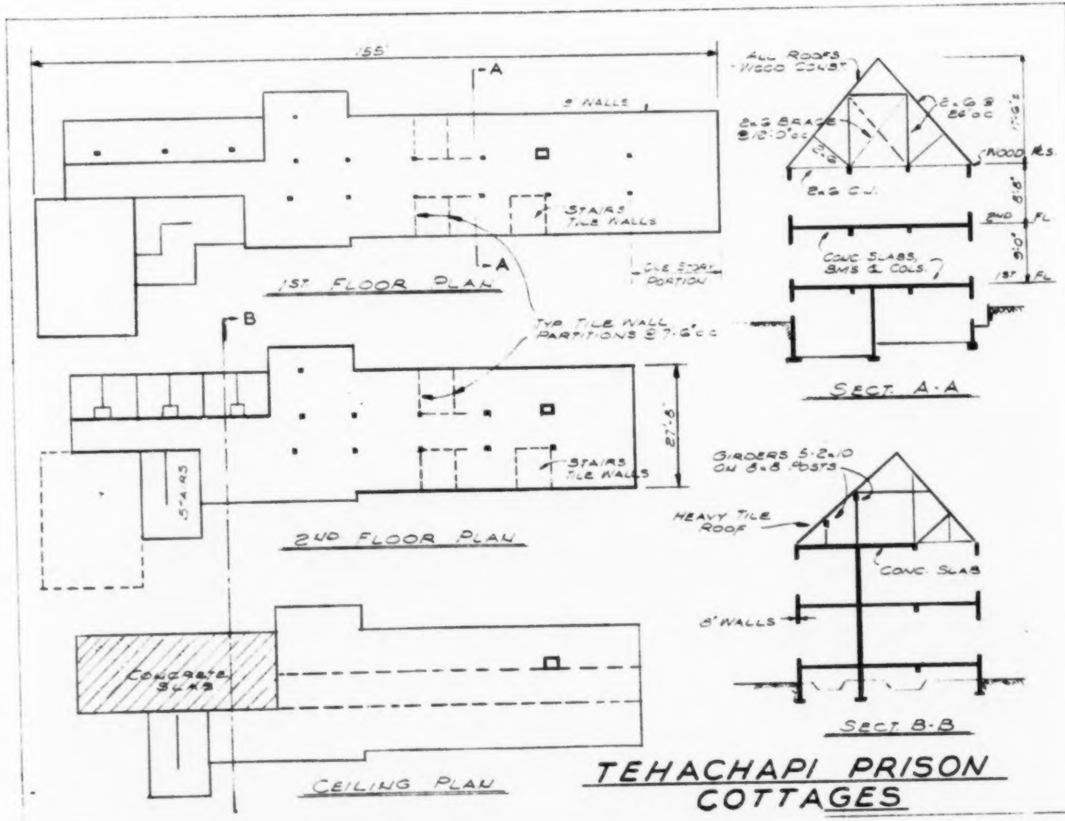

Fig. 13 Framing for typical Cottage at Tehachapi State Prison.

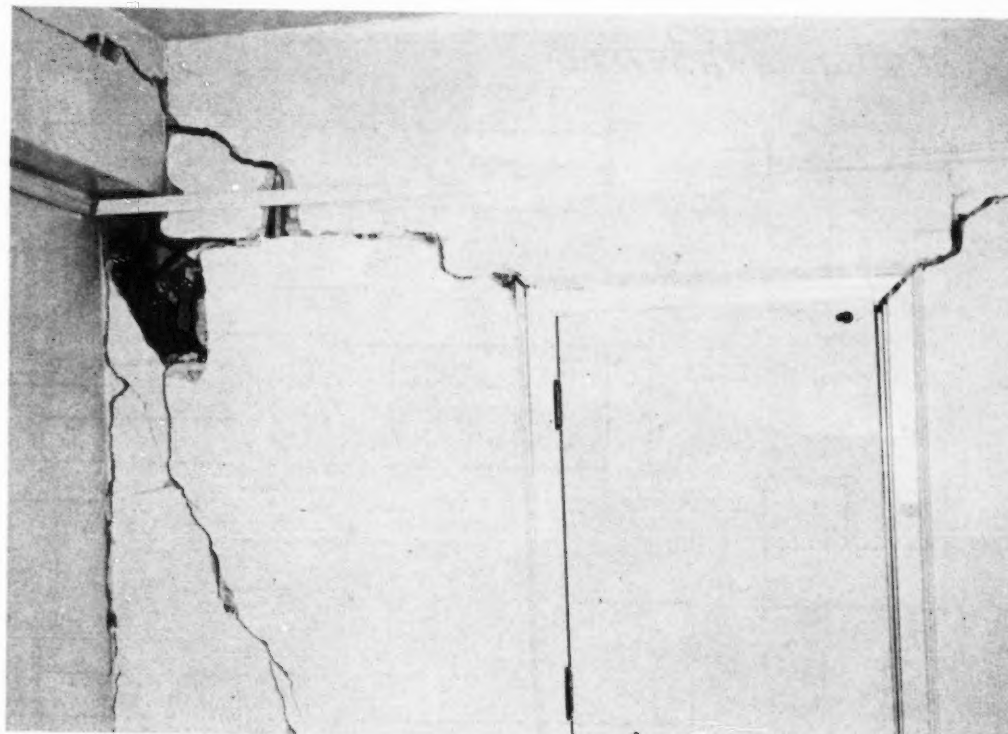

Fig. 14 Tehachapi State Prison. Interior tile partitions in top floors were badly cracked.

Fig. 15. Exterior of Arvin High School.

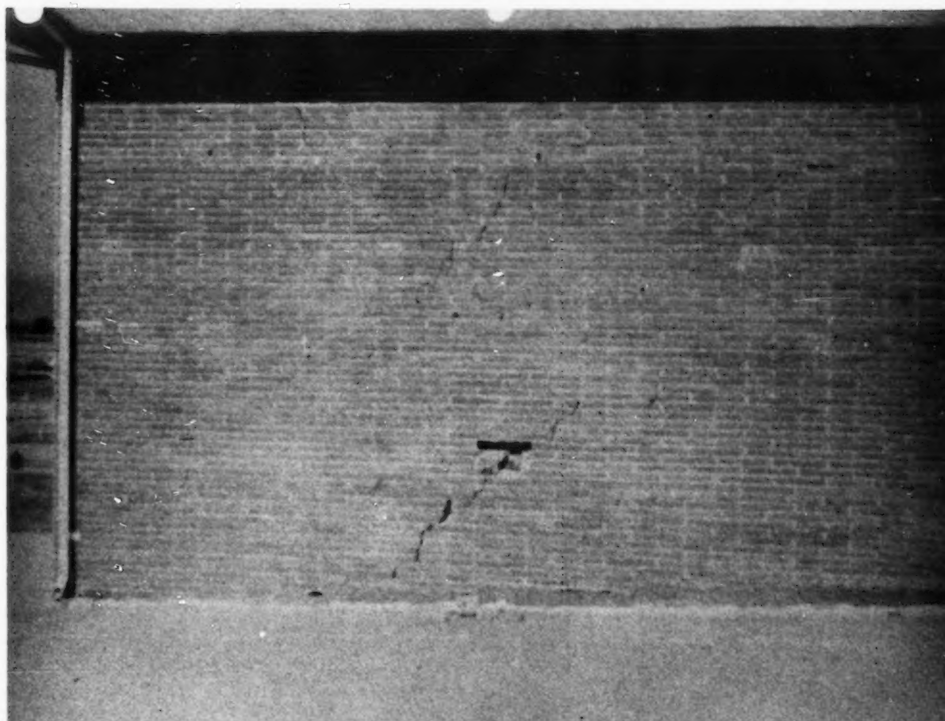

Fig. 16. Arvin High School. This grouted brick shear wall suffered a 1/4" wide crack. Calculated shear stresses were low but the quality of workmanship on this wall was questionable.

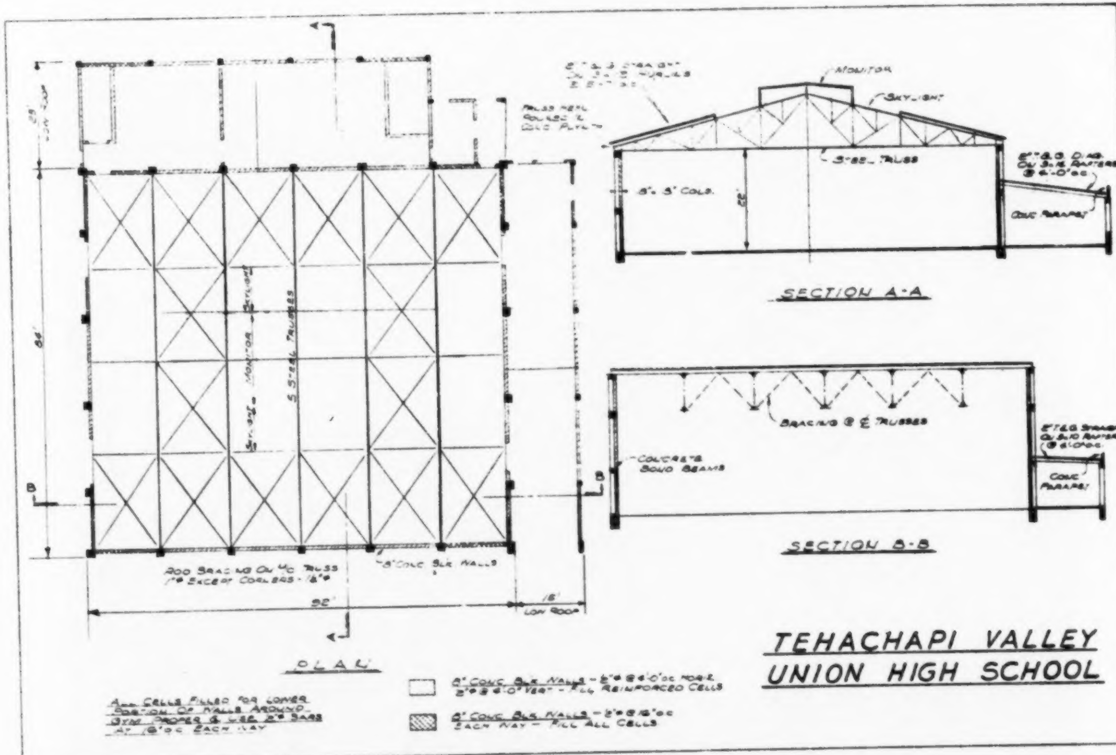

Fig. 17. Framing for Tehachapi Valley Union High School. Reinforced concrete block walls with wood roof on steel roof trusses.

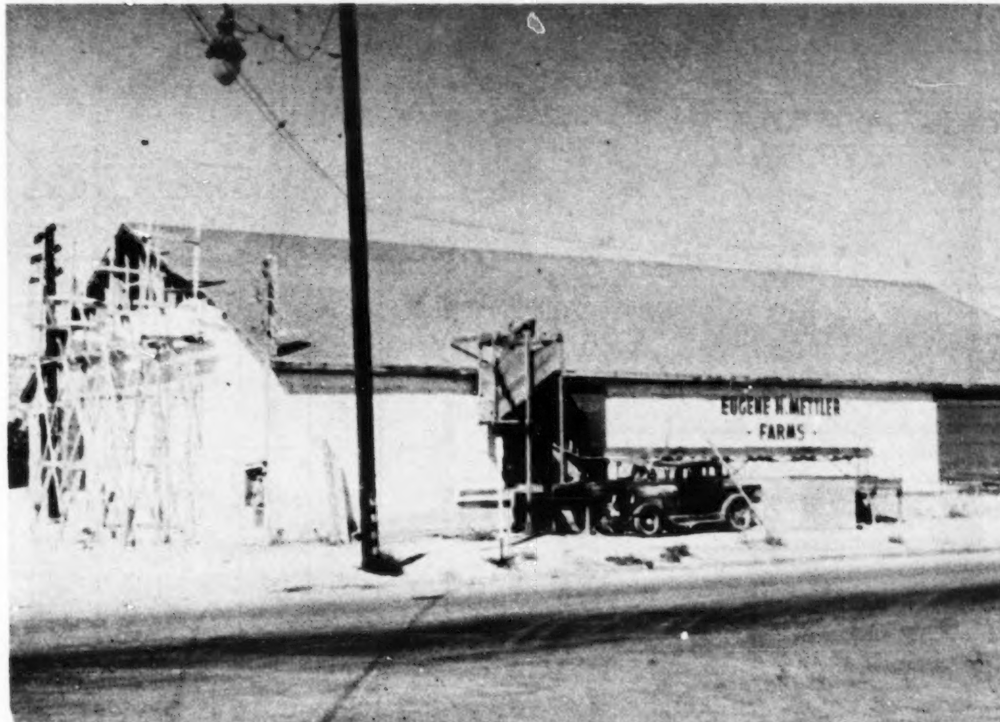

Fig. 18. Concrete block warehouse, Tehachapi.

Fig. 19. Concrete block warehouse end gables were thrown out. There was no connection to roof framing.

Fig. 20. Tehachapi Supply Co. Concrete block building with reinforced concrete columns and bond beams.

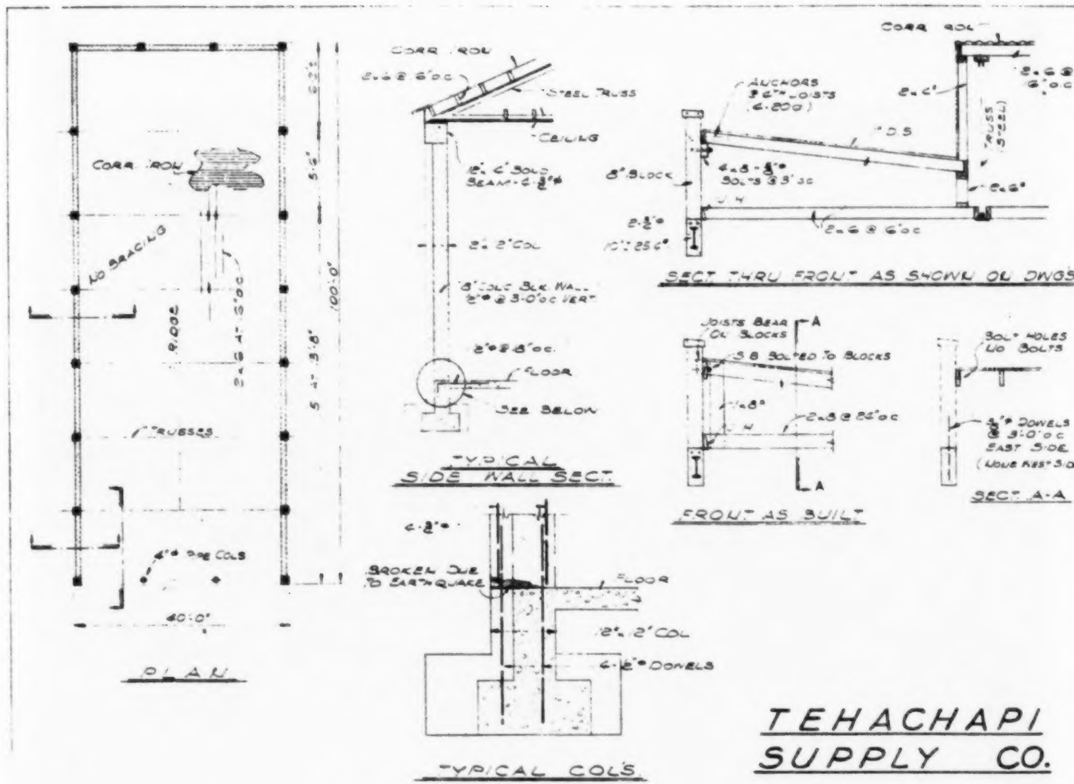

Fig. 21. Framing for Tehachapi Supply Co.

Fig. 22. Broken column base at Tehachapi Supply Co. Bars shown are 1/2"Ø dowels to 3/4"Ø column bars above. Concrete quality of footings was of poor quality.

Fig. 23. Two story brick building in Tehachapi. The second floor was used as a lodge assembly room. When the walls fell out, the roof fell in, here shown supported on the seats and a grand piano.

Fig. 24. Bank of Tehachapi. Reinforced concrete walls, with wood and stucco front and concrete block parapets.

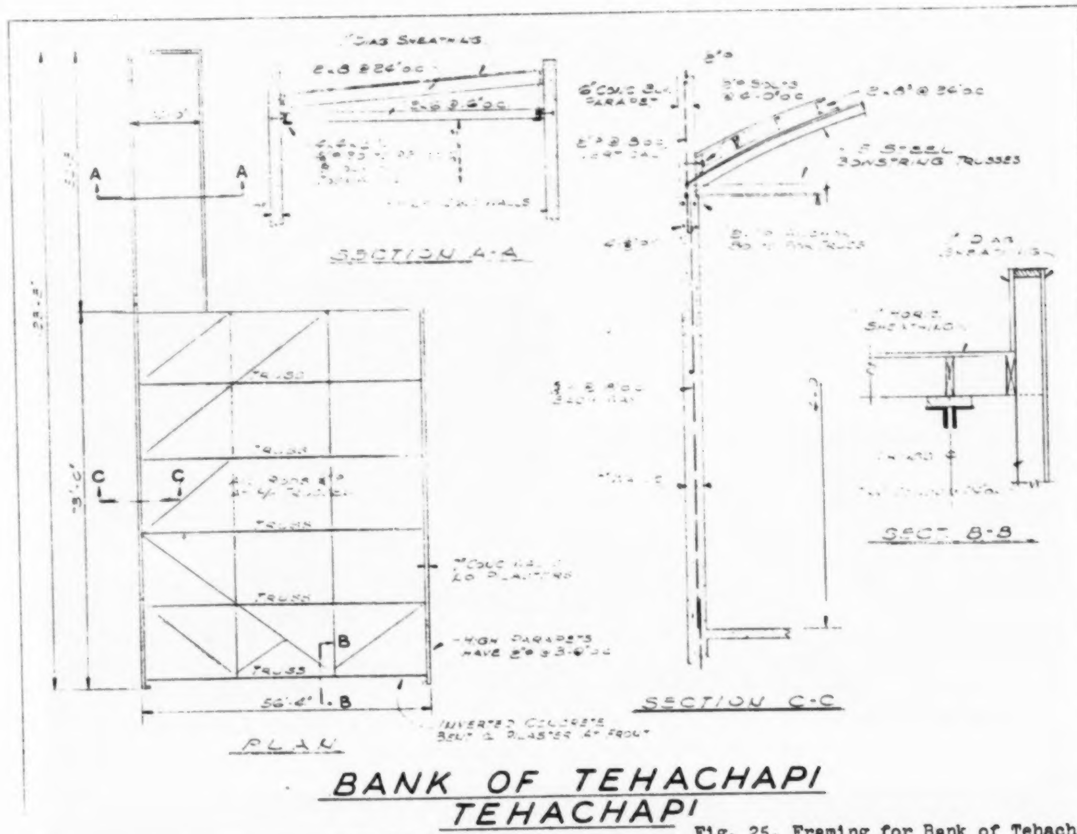

Fig. 25. Framing for Bank of Tehachapi.

Fig. 26. Bank of Tehachapi. Stucco front was cracked and high parapet cracked (opposite side fell off) due to lack of reinforcement lap in concrete blocks.

Fig. 27. Bekay Theater, Tehachapi. This reinforced concrete building with a wood roof was undamaged.

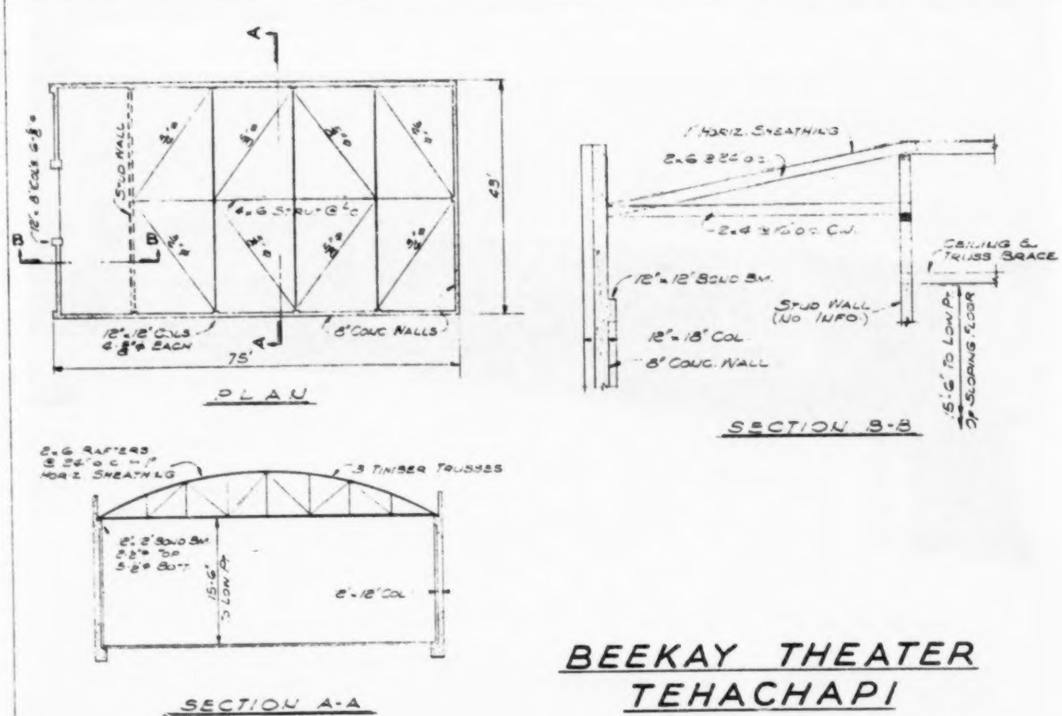

Fig. 28. Framing for Bekay Theater.

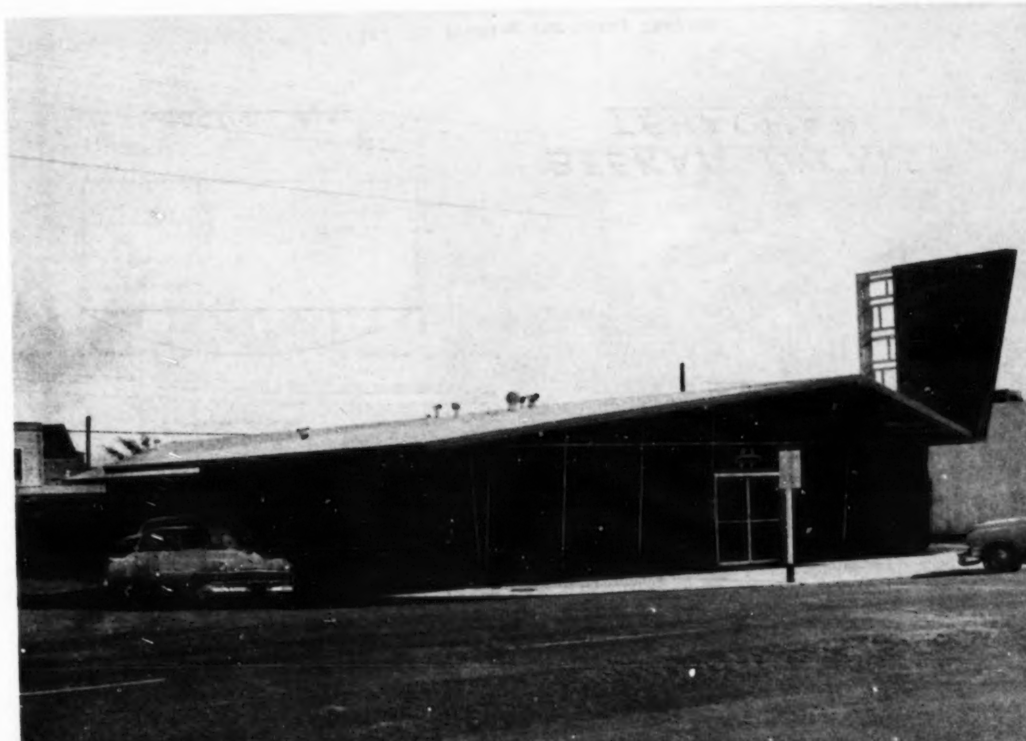

Fig. 29. Simpson Motors, Arvin. This modern building, well designed and closely supervised was essentially undamaged.

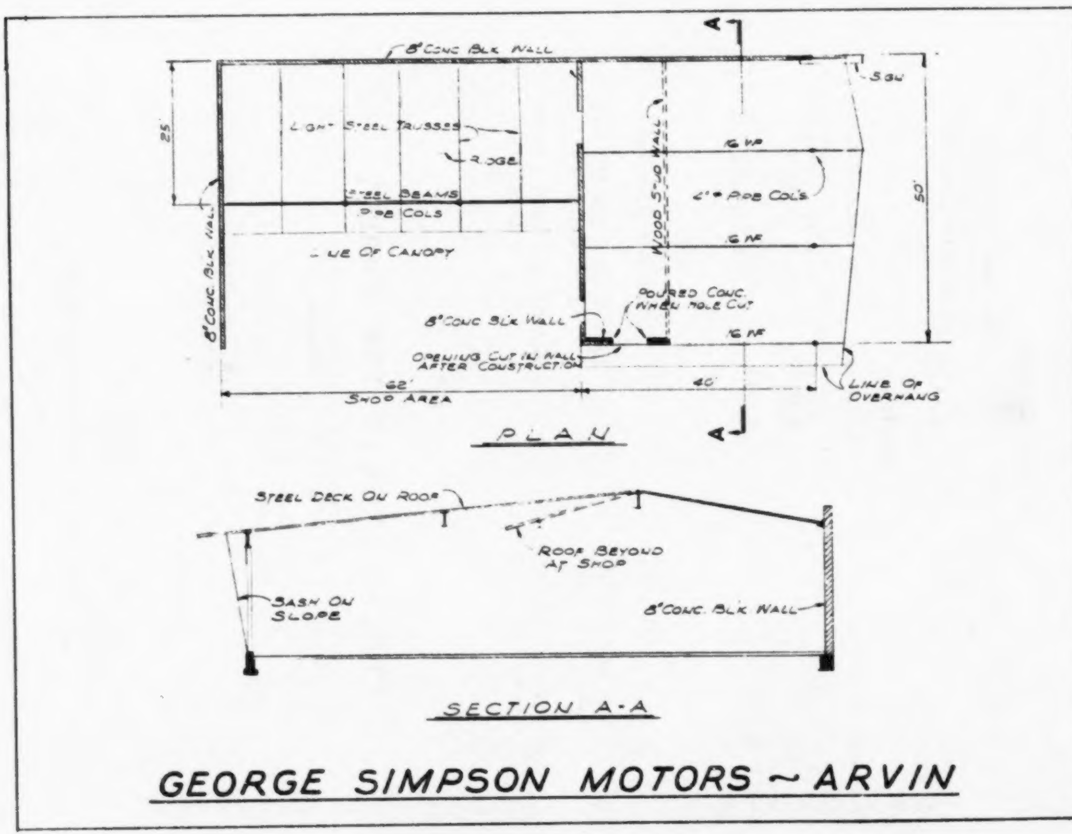

Fig. 30. Framing for Simpson Motors.

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